(b) Modification factors, m, for enhanced performance objectives and linear elastic procedures, are provided in Table 7-7 for deformation-controlled reinforced masonry in-plane walls and piers.

(c) Modeling parameters and numerical acceptance criteria for nonlinear procedures applied to deformation-controlled reinforced masonry walls and piers are provided in Table 7-8.

(d) Expected strength. The expected lateral strength of reinforced masonry, Q_{CE}, in either flexure or shear shall be determined using 1.25 f_y for the contribution attributed to the reinforcement.

(e) Lower-bound strength. The lower-bound strength for all other actions in URM or reinforced masonry shear walls shall be taken as the design strength defined by Section 11.5.3 of FEMA 302.

i. Wood Stud Shear Walls.

(1) General design criteria. The criteria used to design wood stud shear walls are presented in Chapter 12 of FEMA 302. Additional criteria and details are included in the following paragraphs.

(2) Allowable shears for plywood. Details of plywood sheathed walls are shown in Figure 7-17, and the allowable shears are shown in Table 7-9. When a combination of plywood and other materials is used, the shear strength of the walls will be determined by the values permitted for plywood alone.

(3) Conventional light frame construction, as defined in Section 12.5 of FEMA 302, may be used only for buildings required to comply with Performance Objective 1A.

(4) Deflections. Procedures for calculating the deflection of wood frame shear walls are not yet available. The maximum height-width limitations given herein are presumed to satisfactorily control deflections. Relative stiffness of wood stud shear walls will be measured by the effective lineal width of walls or piers between openings.

(5) Wall tie-down. The end studs of any plywood sheathed shear wall and/or shear wall pier will be tied down in such a manner as to resist the overturning forces produced by seismic forces parallel to the shear wall. This overturning force is sometimes of sufficient magnitude to require special steel attachment details. A commonly used detail is shown on Figure 7-18.

(6) Acceptance criteria.

(a) Compliance with the provisions of FEMA 302 constitutes the acceptance criteria for Performance Objective 1A for light frame construction.

(b) Response modification factors, R, for Performance 1A are provided in Table 7-1 for light frame walls in bearing-wall systems and building frame systems.

f_{ae}/f_{me}	L/h_{eff}	$\rho_g f_{ye}/f_{me}$	m Factors					
			Primary				Secondary	
			IO	SE	LS	CP	LS	CP
0.00	0.5	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	2.5	3.8	5.0	6.5	8.0	10.0
		0.20	1.5	1.8	2.0	2.5	4.0	5.0
	1.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	3.5	5.0	6.5	7.5	8.0	10.0
		0.20	1.5	2.3	3.0	4.0	6.0	8.0
	2.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	3.5	5.0	6.5	7.5	8.0	10.0
		0.20	2.0	2.8	3.5	4.5	7.0	9.0
0.038	0.5	0.01	3.0	4.5	6.0	7.5	8.0	10.0
		0.05	2.0	3.3	3.5	4.5	7.0	9.0
		0.20	1.5	1.8	2.0	2.5	4.0	5.0
	1.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	2.5	3.8	5.0	6.5	8.0	10.0
		0.20	1.5	2.0	2.5	3.5	5.0	7.0
	2.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	3.5	5.0	6.5	7.5	8.0	10.0
		0.20	1.5	2.8	3.0	4.0	6.0	8.0
0.075	0.5	0.01	2.0	3.8	3.5	4.5	7.0	9.0
		0.05	1.5	2.3	3.0	4.0	6.0	8.0
		0.20	1.0	1.5	2.0	2.5	4.0	5.0
	1.0	0.01	2.5	3.8	5.0	6.5	8.0	10.0
		0.05	2.0	2.8	3.5	4.5	7.0	9.0
		0.20	1.5	2.0	2.5	3.5	5.0	7.0
	2.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	2.5	3.8	5.0	6.5	8.0	10.0
		0.20	1.5	2.3	3.0	4.0	4.0	8.0

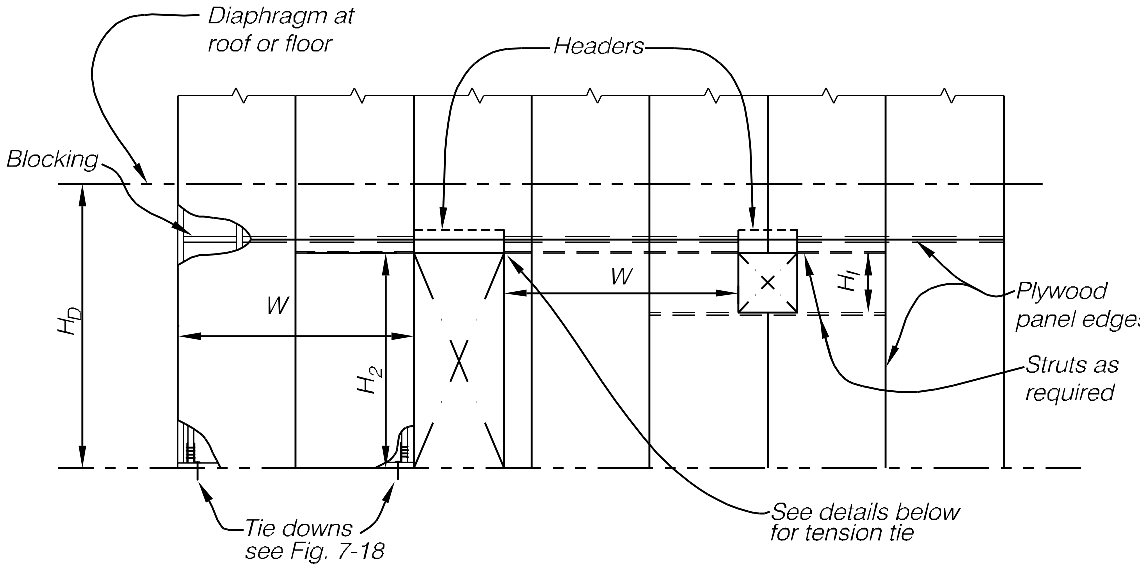
Note: Interpolation is permitted between table values.

Table 7-7: Linear Static Procedure—m Factors for Reinforced Masonry In-Plane Walls

f_{ae}/f_{me}	L/h_{eff}	$\rho_g f_{ye}/f_{me}$	c	d %	e %	Acceptance Criteria					
						Primary				Secondary	
						IO %	SE %	LS %	CP %	LS %	CP %
0.00	0.5	0.01	0.5	2.6	5.3	1.0	1.5	2.0	2.6	3.9	5.3
		0.05	0.6	1.1	2.2	0.4	0.6	0.8	1.1	1.6	2.2
		0.20	0.7	0.5	1.0	0.2	0.3	0.4	0.5	0.7	1.0
	1.0	0.01	0.5	2.1	4.1	0.8	1.2	1.6	2.1	3.1	4.1
		0.05	0.6	0.8	1.6	0.3	4.5	0.6	0.8	1.2	1.6
		0.20	0.7	0.3	0.6	0.1	0.2	0.2	0.3	0.5	0.6
	2.0	0.01	0.5	1.6	3.3	0.6	0.9	1.2	1.6	2.5	3.3
		0.05	0.6	0.6	1.3	0.2	0.4	0.5	0.6	0.9	1.3
		0.20	0.7	0.2	0.4	0.1	0.2	0.2	0.2	0.3	0.4
0.038	0.5	0.01	0.4	1.0	2.0	0.4	0.6	0.8	1.0	1.5	2.0
		0.05	0.5	0.7	1.4	0.3	0.4	0.5	0.7	1.0	1.4
		0.20	0.6	0.4	0.9	0.2	0.3	0.3	0.4	0.7	0.9
	1.0	0.01	0.4	0.8	1.5	0.3	0.5	0.6	0.8	1.1	1.5
		0.05	0.5	0.5	1.0	0.2	0.3	0.4	0.5	0.7	1.0
		0.20	0.6	0.3	0.6	0.1	0.2	0.2	0.3	0.4	0.6
	2.0	0.01	0.4	0.6	1.2	0.2	0.3	0.4	0.6	0.9	1.2
		0.05	0.5	0.4	0.7	0.1	0.2	0.3	0.4	0.5	0.7
		0.20	0.6	0.2	0.4	0.1	0.1	0.1	0.2	0.3	0.4
0.075	0.5	0.01	0.3	0.6	1.2	0.2	0.4	0.5	0.6	0.9	1.2
		0.05	0.4	0.5	1.0	0.2	0.3	0.4	0.5	0.8	1.0
		0.20	0.5	0.4	0.8	0.1	0.2	0.3	0.4	0.6	0.8
	1.0	0.01	0.3	0.4	0.9	0.2	0.3	0.3	0.4	0.7	0.9
		0.05	0.4	0.4	0.7	0.1	0.2	0.3	0.4	0.5	0.7
		0.20	0.5	0.2	0.5	0.1	0.2	0.2	0.2	0.4	0.5
	2.0	0.01	0.3	0.3	0.7	0.1	0.2	0.2	0.3	0.5	0.7
		0.05	0.4	0.3	0.5	0.1	0.2	0.2	0.3	0.4	0.5
		0.20	0.5	0.2	0.3	0.1	0.1	0.1	0.2	0.2	0.3

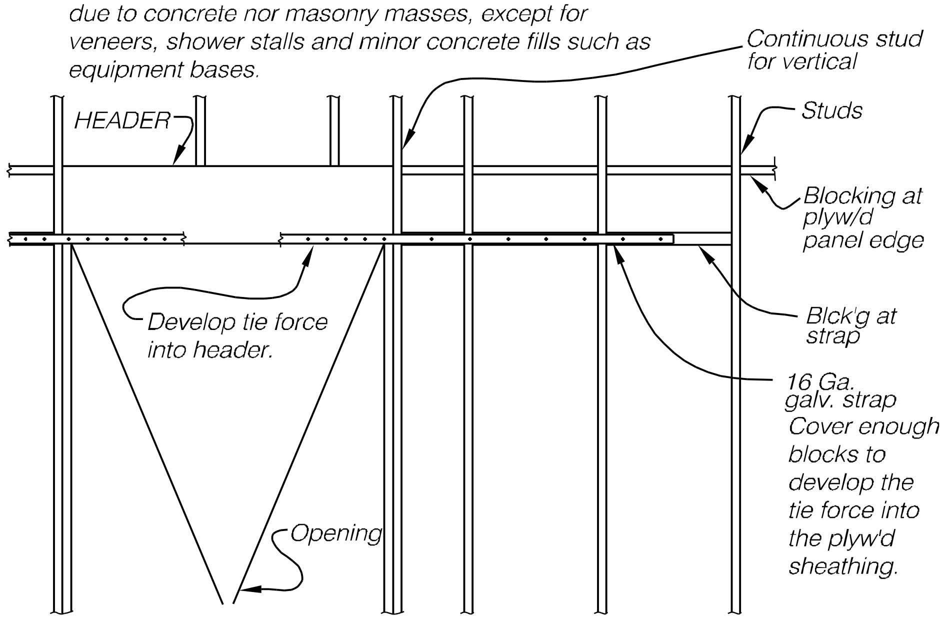
Note: Interpolation is permitted between table values.

Table 7-8: Nonlinear Static Procedure—Simplified Force-Deflection Relations for Reinforced Masonry Shear Walls



<u>NOTES:</u>

1. For values of plywood sheathed shear walls, see Table 7-10.
2. Height-width ratio of plywood sheated shear walls will be limited to 3-1/2 to 1. H_D / w will be used for the height-width ratio unless struts are developed at the top and bottom of the openings in which case H_1 / w or H_2 / w may be used.
3. These shear walls shall not be used to resist forces due to concrete nor masonry masses, except for veneers, shower stalls and minor concrete fills such as equipment bases.



Detail of tension tie-strut

Figure 7-17 Plywood sheathed wood stud shear walls.

TABLE 7-9 Factored Shear Resistance in Kips per Foot (KLF) for Seismic Forces on Structural Use Panel Shear Walls with Framing Members of Douglas Fir-Larch or Southern Pine

Panel Grade	Nail Size (Common or Hot-Dipped Galvanized Box)	Minimum Penetration in Framing (in.)	Panel Thickness (in.)	Panel Applied Direct to Framing Nail Spacing at Panel Edges (in.)				Nail Size (Common or Hot-Dipped Galvanized Box)	Panel Applied Over ½ in. or 5/8 in. Gypsum Sheathing Nail Spacing at Panel Edges (in.)			
				6	4	3	2[d]		6	4	3	2[d]
Structural 1	6d	1¼	3/8	0.23	0.35	0.46	0.60	8d	0.23	0.35	0.46	0.60
	8d	1½	3/8	0.27[f]	0.42	0.54[f]	0.71[f]	10d[e]	0.27[f]	0.42[f]	0.54[f]	0.71[f]
	8d	1½	7/16	0.30[f]	0.46[f]	0.59[f]	0.78[f]	10d[e]	0.30[f]	0.46[f]	0.59[f]	0.78[f]
	8d	1½	15/32	0.33	0.50	0.64	0.85	10d[e]	0.33[f]	0.50[f]	0.64[f]	0.85[f]
	10d[e]	1-5/8	15/32	0.40	0.60	0.78	1.02		-	-	-	-
	14 ga staple	2	3/8	0.17	0.26	0.35	0.52		-	-	-	-
	14 ga staple	2	7/16	0.24	0.36	0.48	0.72		-	-	-	-
Sheathing, Panel Siding and Other Grades Covered in References 9.10 and 9.11	6d	1¼	3/8	0.23	0.35	0.46	0.60	8d	0.23	0.35	0.46	0.60
	8d	1½	3/8	0.26[f]	0.37[f]	0.48[f]	0.62[f]	10d[e]	0.26[f]	0.37[f]	0.48[f]	0.62[f]
	8d	1½	7/16	0.28[f]	0.41[f]	0.53[f]	0.68[f]	10d[e]	0.28[f]	0.41[f]	0.53[f]	0.68[f]
	8d	1½	15/32	0.30	0.44	0.57	0.75	10d[e]	0.30[f]	0.44[f]	0.57[f]	0.75[f]
	10d[e]	1-5/8	15/32	0.36	0.54	0.70	0.90		-	-	-	-
	10d[e]	1-5/8	19/32	0.40	0.60	0.78	1.02		-	-	-	-
	14 ga staple	2	3/8	0.15	0.23	0.30	0.45		-	-	-	-
	14 ga staple	2	7/16	0.21	0.32	0.42	0.63		-	-	-	-
	14 ga staple	2	15/32	0.24	0.36	0.48	0.72		-	-	-	-
	(Hot-Dipped Galvanized Casing Nail)							(Hot-Dipped Galvanized Casing Nail)				
Panel Siding as Covered in Reference 9.10	6d	1¼	3/8	0.16	0.25	0.32	0.42	8d	0.16	0.25	0.32	0.42
	8d	1½	3/8	0.19	0.28	0.36	0.48	10d[e]	0.19	0.28	0.36	0.48

TABLE 7-9 (cont.)

Factored Shear Resistance in Kips per Foot (KLF) for Seismic Forces on Structural Use Panel Shear Walls with Framing Members of Douglas Fir-Larch or Southern Pine

a $\lambda = 1.0$ $\varnothing = 0.65$

b All panel edges backed with 2-inch nominal or wider framing. Panels installed either horizontally or vertically. Space nails at 6 inches on center along intermediate framing members for 3/8-inch panels installed with strong axis parallel to studs spaced 24 inches on center and 12 inches on center for other conditions and panel thicknesses. Allowable shear values for fasteners in framing members of other species set forth in Table 12A of ASCE 16-95 shall be calculated for all grades by multiplying the values for fasteners in STRUCTURAL I by the following factors: 0.82 for species with a specific gravity greater than or equal to 0.42 but less than 0.49 ($0.42 \leq G < 0.49$) and 0.65 for species with a specific gravity less than 0.42 ($G < 0.42$). For panel siding using hot-dipped galvanized casing nails, the shear values shall be the values in the table multiplied by the same factors.

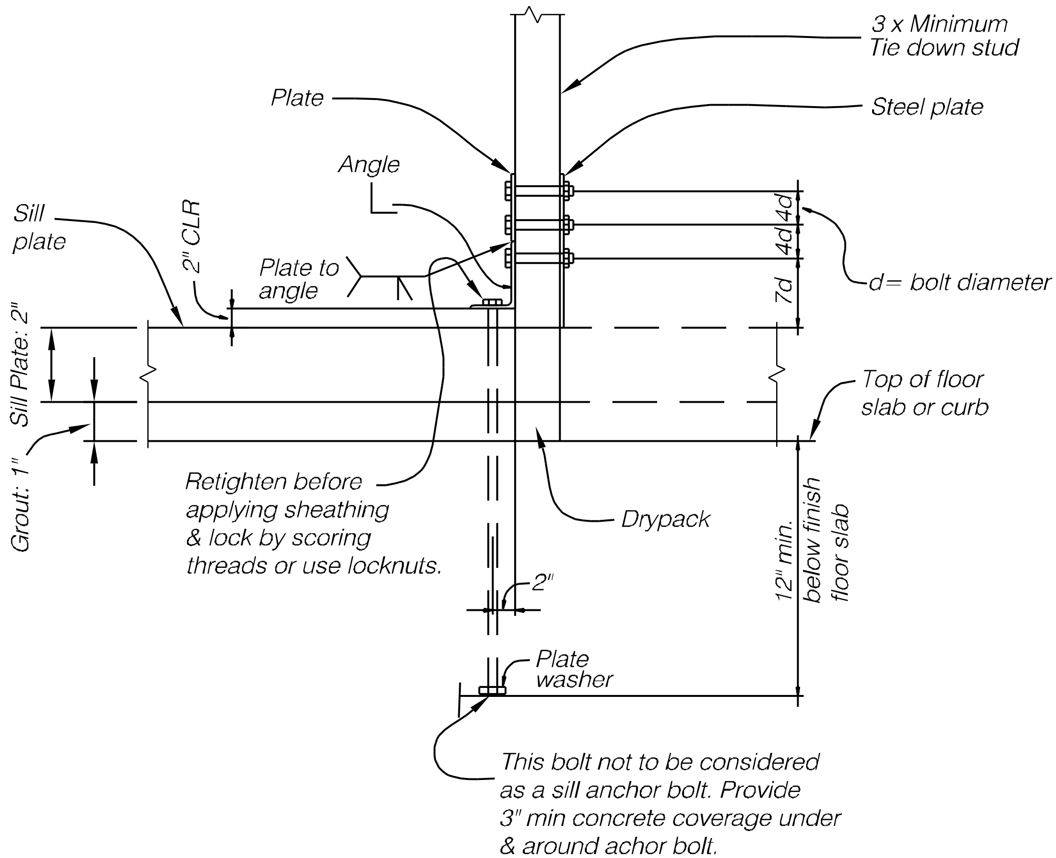
c Where panels are applied on both faces of a wall and nail spacing is less than 6 inches on center on either side, panel joints shall be offset to fall on different framing members, or framing shall be 3-inch nominal or wider, and nails on each side of joint shall be staggered.

d Framing at adjoining panel edges shall be 3-inch nominal or wider, and nails shall be staggered where nails are spaced 2 inches on center.

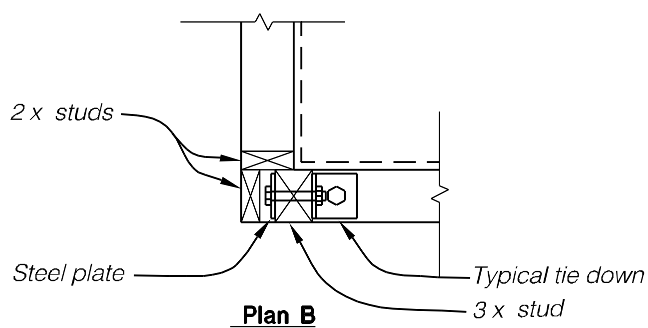
e Framing at adjoining panel edges shall be 3-inch nominal or wider, and nails shall be staggered where 10d nails having penetration into framing of more than 1-5/8 inches are spaced 3 inches or less on center.

f The values for 38-inch and 7/16-inch panels applied directly to framing are permitted to be increased to the values shown for 15/32-inch panels, provided studs are spaced a maximum of 16 inches on center or panel is applied with strong axis across studs.

Metric equivalent:
1 inch = 25mm
1 kLF = 14.6 kN/m



Typical tie-down detail A



Plan B

Corner detail at tie-down

1 inch = 25mm

Figure 7-18 Wood stud walls - typical tie-down details

j. Steel Stud Shear Walls.

(1) Description of system. Steel studs may be used in lieu of wood studs in structural bearing walls. To function as shear walls, steel-stud walls need bracing. In principle, plywood sheathing could be used, but there are no available allowable shear values. Instead, it is customary to use diagonal braces made of steel straps welded to the face of the steel studs. Sheathing such as plywood or gypsum board may be used to serve architectural purposes such as containing insulation and backing up finishes.

(2) Design Criteria. The Department of Defense is currently reviewing tests performed by industry with the objective of providing approved design criteria for steel stud framing systems. A moratorium currently precludes the use of this system as a lateral-force-resisting system. It is anticipated that applicable criteria will be available prior to the final version of this document.

7-3. Steel Braced Frames.

a. General.

(1) Function. Vertical braced frames are used to transmit lateral forces from the diaphragm above to the diaphragm below or to the foundations. They are similar to shear walls in their general function and stiffness is compared with moment-resisting frames.

(2) Definition of braced frame. A braced frame is defined as an essentially vertical truss system of the concentric or eccentric type that is provided to resist lateral forces. Note that for braced frames, as for shear walls, the *R* value depends on whether the frame is in a building-frame system, a moment-resisting frame system, or a dual system.

(3) Redundancy. A sufficient number of braced frames should be provided so that a failure of a single member or connection will not result in instability of the entire lateral-force-resisting system.

(4) Braced frame types. The principal types of braced frame are the familiar concentric braced frame (CBF), the relatively new eccentric braced frame (EBF), and the knee-braced frame (KBF).

(5) Design criteria. The criteria governing the design of structural steel and wood vertical braced frames will be as prescribed in this chapter. Reinforced concrete braced frames are not permitted in buildings governed by this document.

(a) Structural steel braced frames. Structural steel braced frames will conform to the requirements of the AISC "Seismic Provisions for Structural Steel Buildings" and the further provisions of this document.

(b) Wood braced frames. Wood braced frames will be designed by using normal procedures illustrated in many easily obtainable texts and are not covered in this manual. Allowable loads and resistance factors for wood members shall be in accordance with ASCE 16-95.

b. Concentric Braced Frames.

(1) Eccentricities. Although the frame is called "concentric," there may be minor eccentricities between member centerlines at the joints, and these

eccentricities must be provided for in the design. Such eccentricities do not mean that the frame is an EBF: the EBF has unique properties and design methods.

(2) Concentric braced frame types. Braced frames are usually of steel and may be of various forms. Figure 7-19 illustrates some of the common configurations for concentric braced frames. Trussed portal bracing and K-bracing sometimes used in the older industrial buildings are still occasionally used in bridge design, but have been replaced in buildings by one or more of the configurations shown in Figure 7-19. Braced frames with single diagonal members capable of resisting compression as well as tension are used to permit flexibility in the location of openings. Chevron bracing also permits openings in the middle of the braced bay, but the horizontal beam at bracing intersection must be capable of resisting an additional load equal to the vertical component of the tensile brace when the compressive brace buckles. For all of the bracing configurations in Figure 7-19, the deflection of the braced frame is readily computed using recognized methods.

(3) Direction of brace force. Braces that are designed for compression will, of course, act also in tension. Diagonal members designed to resist both compression and tension forces are preferred because they provide greater system redundancy. X-braced panels are the most effective bracing configurations as the tension diagonal provides direct in-plane lateral support to the compression diagonal and also provides out-of-plane resistance to compression buckling (as indicated in Figure 7-21 an unbraced length equal to two-thirds of the total length of the compression brace may be used for the effective out-of-plane length). Braces may be designed for tension only, but the use of such braces is discouraged because they tend to stretch under earthquake tension, go slack during the load reversal, then snap when tension is applied in a subsequent cycle. Diagonal cable bracing is permitted only for utilitarian one-story Seismic Use Group I buildings in areas with $S_{DS} \leq 0.50$g, and where the system is not required to provided lateral support for concrete or masonry walls.

(4) Effect of bracing on columns. The vertical component of brace force is transferred into the column, and adds to or subtracts from the gravity load on the column. When braces are few and heavily loaded, their vertical components may govern the design of the columns. The concern with braces of this type is that their true, as-built ultimate capacity may be greater than is assumed in design, and therefore, that such braces could overload the column to the point of collapse.

(5) Configurations. Diagonal X-bracing is the preferred configuration in that the tension brace can provide in-plane lateral support to the compression brace. The orientation of single braces should be alternated so that not all of the braces are in tension or compression at the same time. Chevron bracing may have an interaction with gravity-load-carrying beams; accordingly, special requirements are provided in the AISC Seismic Provisions. K-bracing has a potentially dangerous effect on columns;

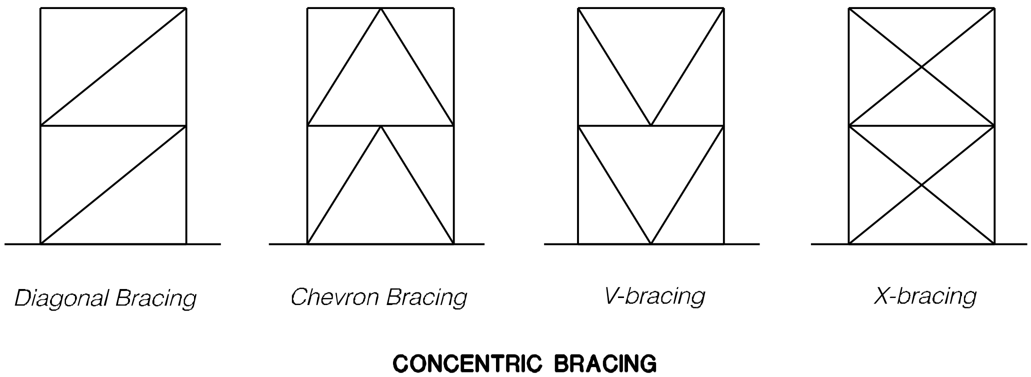


Figure 7-19 Concentric braced frames.

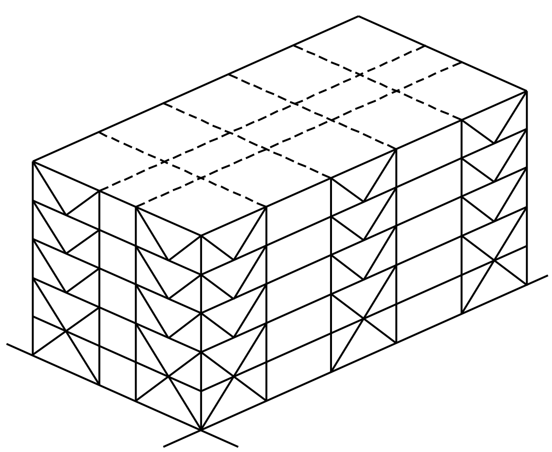


Figure 7-20 Bracing for a tier building.

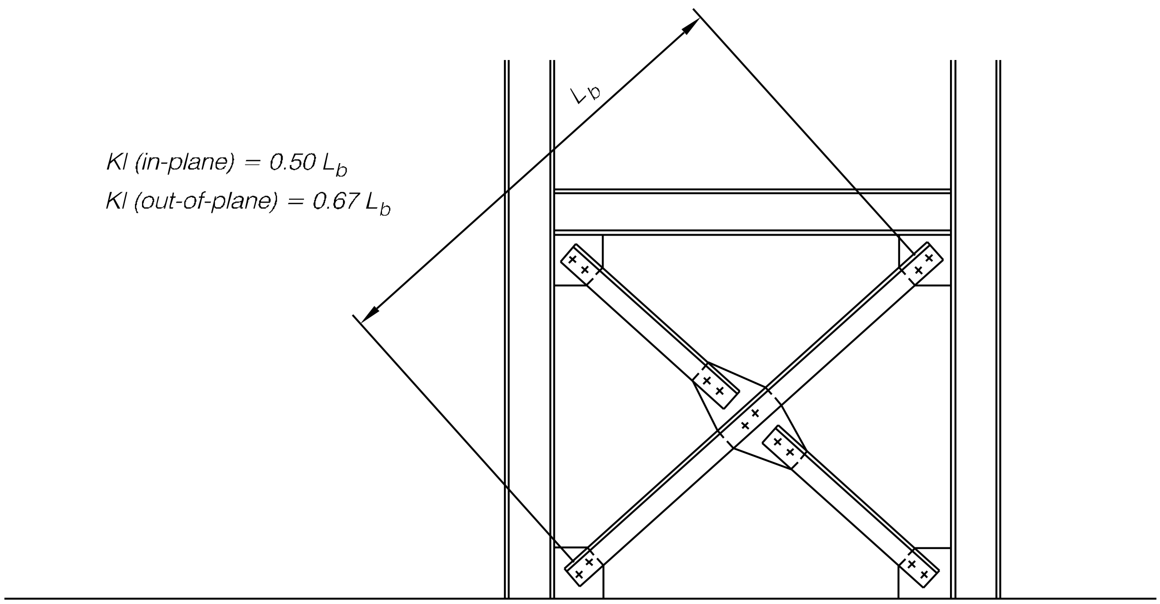


Figure 7-21 Effective length of cross-bracing.

accordingly, it is subject to the requirements of Section 14.4b, Part I, of the AISC Seismic Provision, and permitted only in buildings in Seismic Design Categories A and B.

(6) Low buildings. The AISC Seismic Provisions provide special provisions for concentric bracing in metal buildings not over two stories, and for light roof structures such as penthouses. Manufactured metal buildings are intended to be included in this category. In planning the use of manufactured metal buildings, the designer is cautioned that these buildings can perform well only when they are kept light and simple, as they are intended to be; they may have poor performance if extra weight, such as masonry veneer, is added, or if they are used as elements of a more complex system.

(7) Knee-braced frames (KBF).

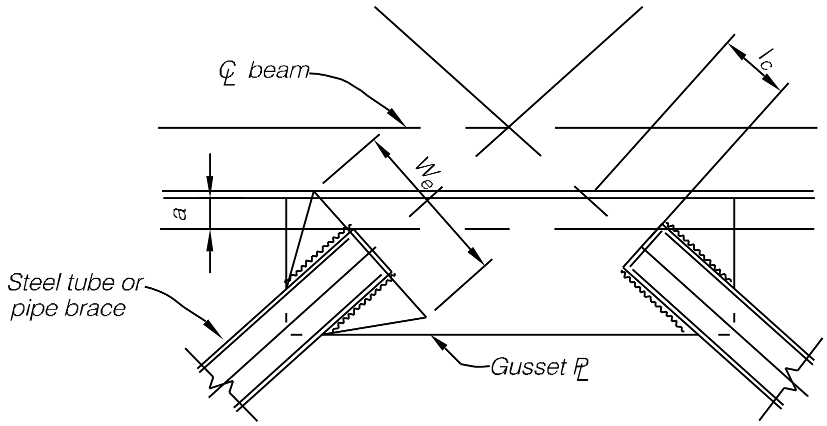
(a) Definition. A KBF is an assembly of a beam, a column, and a brace whose ends are significantly offset from the beam-column joints. The braces in CBFs are either truly concentric, or have small eccentricities with the beam-column joints; accordingly, they induce forces that are primarily axial, while the braces in KBFs have substantial eccentricities, and induce significant shearing, and flexural, as well as axial, stresses in the columns and beams.

(b) Function. Knee braces were often used in the past to stiffen beams and to provide a measure of lateral stability. Their popularity in recent years has decreased markedly, particularly in zones of high seismicity, because their seismic behavior has become recognized as potentially dangerous.

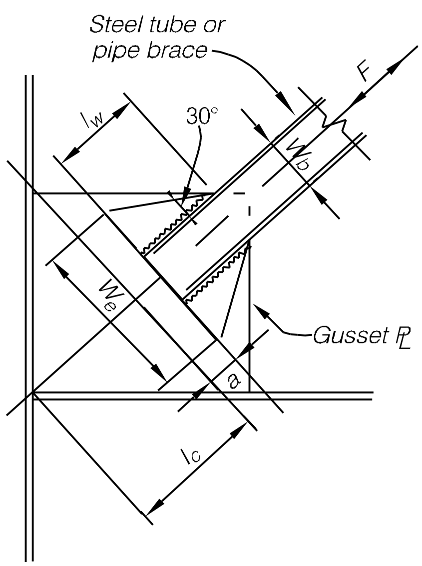
(c) Design considerations. There are two concerns with KBFs. The first concern involves gravity load: any change in the load on the beam after the brace is connected induces forces in all the components of the frame; moreover, the brace has a prying effect that can produce surprisingly large forces in the beam-column joint. The sequence of erection and the further application of superimposed loads must be carefully controlled. The second concern involves seismic loads: another set of loads is applied, and while the brace does stiffen the frame, its as-built ultimate capacity may cause bending in the column of sufficient magnitude to cause collapse.

(d) Design criteria. KBFs shall be designed in accordance with Section 9.4 of the AISC Seismic Provisions, and the use of KBFs shall be restricted to roof structures or to unoccupied storage or other utilitarian buildings with Performance Objective 1A, not over two stories in height.

(8) Connections. The AISC Seismic Provisions provide the requirements for design of connections. Figure 7-22 illustrates the design of gusset plates with welded connections. Note that most steel braces are designed as pin-ended members (K=1.0) for compressive forces. As the braces deflect out-of-plane in compression, the gusset must be able to accommodate the end-rotation. The AISC LRFD Specifications prescribe that the brace connection should provide a minimum length of gusset plate, a, equal to twice the plate thickness, t, to permit end-rotation of the brace as shown in Figure



DETAIL 1



DETAIL 2

Gusset Plate Design Criteria:

Max brace force F:
- *Tension:* $= R_y F_y A_g$
- *Compr.* $= A_g F_{cr}$

Shear/tension rupture:
- $0.75t\,(0.6\,F_y \times 2\,l_w/\cos 30^\circ + F_u \times W_e)$ *or*
- $0.75t\,(0.6\,F_u \times 2\,l_w + F_y \times W_b)$

Tension:
- $0.75 \times W_e \times F_u \times t \geq R_y F_y A_g$

Compression:
- $0.90 \times F_y \times W_e \times t \geq A_g F_{cr}$

Buckling:
- $0.90 \times \dfrac{4000\,t^3\sqrt{F_y}}{l_c} \geq A_g F_{cr}$

Rotation capability:
- $a \geq 2\,t$

Figure 7-22 Gusset plate design criteria.

7-22. When the in-plane brace connection is welded as shown in Figure 7-21, the appropriate *K* value for restrained end conditions should be used, and the welds and gusset plate should be designed for the plastic moment capacity of the brace. For the gusset plate, a section, normal to the brace, or the midpoint of the connection, should have the necessary capacity to resist the above moment.

(9) Acceptance criteria.

(a) Response modification factors, *R*, for Performance Objective 1A are provided in Table 7-1. K-braced frames shall be classified as ordinary concentric braced frames and are subject to the limitations of Paragraph (6) above.

(b) Modification factors, *m,* for enhanced performance objective are provided in Table 7-10.

(c) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided on Table 7-11.

(d) The expected strength of deformation-controlled components or elements shall be determined using the expected yield strength, F_{ye}, as defined in the AISC Seismic provisions.

(e) The lower-bound strength of connections and other force-controlled components shall be taken as the nominal strength multiplied by the appropriate resistance factor, N, determined from the provisions of the AISC LRFD Specifications.

c. Eccentric Braced Steel Frames (EBF).

(1) Definition. An EBF is a steel-braced frame designed in accordance with Section 15, Part I, of the AISC Seismic Provisions. At least one end of each brace intersects a beam at a point offset from the beam intersection with the column or with the opposing brace (see Figure 7-23). The short section of the beam between opposing braces, or between a brace and the beam-column intersection, is called the "link beam," and is the element of the frame intended to provide inelastic cyclic yielding.

(2) Purpose. The intent of the eccentric braced frame design is to provide a ductile link that will yield in lieu of buckling of its braces when the frame experiences dynamic loads in excess of its elastic strength. Although they are usually easier to detail, they are more complex to design than CBFs, and they are most useful in areas with S_{DS} $ 0.75.

(3) Characteristics. To take advantage of the ductility of the link, it is important that all related framing elements be strong enough to force the link to yield, and that they maintain their integrity through the range of forces and displacements developed during the yielding of the link. The braces are the most vulnerable of the framing elements because seismic forces are by far the dominant forces in their design. Other elements, such as columns and collector beams, are less vulnerable, since their seismic loads constitute a smaller percentage of their total loads, and since there are frequently redundant load paths for portions of the forces they carry. The rotation demand on the link beam is a multiple of the lateral drift of the frame as a whole, a multiple that is

Component/Action	m Values for Linear Procedures					
	Primary				Secondary	
	IO	SE	LS	CP	LS	CP
Concentric Braced Frames						
Columns:[1]						
a. Columns in compression[1]	Force-controlled member, use Equations 6-4a or 6-4b.					
b. Columns in tension[1]	1	2	3	5	6	7
Braces in Compression[2]						
a. Double angles buckling in plane	0.8	3.4	6	8	7	9
b. Double angles buckling out of plane	0.8	2.9	5	7	6	8
c. W or I shape	0.8	3.4	6	8	6	8
d. Double channel buckling in plane	0.8	3.4	6	8	7	9
e. Double channel buckling out of plane	0.8	2.9	5	7	6	8
f. Rectangular concrete-filled cold-formed tubes	0.8	2.9	5	7	5	7
g. Rectangular cold-formed tubes						
1. $\frac{d}{t} \le \frac{90}{\sqrt{F_y}}$	0.8	2.9	5	7	5	7
2. $\frac{d}{t} \ge \frac{190}{\sqrt{F_y}}$	0.8	1.4	2	3	2	3
3. $\frac{90}{\sqrt{F_y}} \le \frac{d}{t} \le \frac{190}{\sqrt{F_y}}$	Use linear interpolation					
h. Circular hollow tubes						
1. $\frac{d}{t} \le \frac{1500}{F_y}$	0.8	2.9	5	7	5	7
2. $\frac{d}{t} \ge \frac{6000}{F_y}$	0.8	1.4	2	3	2	3
3. $\frac{1500}{F_y} \le \frac{d}{t} \le \frac{6000}{F_y}$	Use linear interpolation					
Braces in Tension[3]	1	4	6	8	8	10
Eccentric Braced Frames						
a. Beams	Governed by link					
b. Braces	Force-controlled, use Equations 6-4a or 6-4b					
c. Columns in compression	Force-controlled, use Equations 6-4a or 6-4b					
d. Columns in tension	1	2	3	5	6	7

Table 7-10: Acceptance Criteria for Linear Procedures—Braced Frames and Steel Shear Walls

	$\frac{\Delta}{\Delta_y}$		Residual Force Ratio	Deformation					
				Primary				Secondary	
Component/Action	***d***	***e***	***c***	**IO**	**SE**	**LS**	**CP**	**LS**	**CP**
Concentric Braced Frames									
a. Columns in compression[1]	Force-controlled, use Equations 6-4a or 6-4b								
b. Columns in tension[1]	6	8	1.000	1	2.5	4	6	7	8
Braces in Compression[2,3]									
a. Two angles buckle in plane	1	10	0.2	0.8	3.4	6	8	8	9
b. Two angles buckle out of plane	1	9	0.2	0.8	2.9	5	7	7	8
c. W or I shape	1	9	0.2	0.8	3.4	6	8	8	9
d. Two channels buckle in plane	1	10	0.2	0.8	3.4	6	8	8	9
e. Two channels buckle out of plane	1	9	0.2	0.8	2.9	5	7	7	8
f. Concrete-filled tubes	1	8	0.2	0.8	2.9	5	7	7	8
g. Rectangular cold-formed tubes									
1. $\frac{d}{t} \le \frac{90}{\sqrt{F_y}}$	1	8	0.4	0.8	2.9	5	7	7	8
2. $\frac{d}{t} \ge \frac{190}{\sqrt{F_y}}$	1	4	0.2	0.8	1.4	2	3	3	4
3. $\frac{90}{\sqrt{F_y}} \le \frac{d}{t} \le \frac{190}{\sqrt{F_y}}$	Use linear interpolation								
h. Circular hollow tubes									
1. $\frac{d}{t} \le \frac{1500}{F_y}$	1	10	0.4	0.8	2.9	5	7	6	9
2. $\frac{d}{t} \ge \frac{6000}{F_y}$	1	4	0.2	0.8	1.4	2	3	3	4
3. $\frac{1500}{F_y} \le \frac{d}{t} \le \frac{6000}{F_y}$	Use linear interpolation								
Braces in Tension	12	15	0.800	1	4.5	8	10	12	14
Eccentric Braced Frames									
a. Beams	Governed by link								
b. Braces	Force-controlled, use Equations 6-4a or 6-4b								
c. Columns in compression	Force-controlled, use Equations 6-4a or 6-4b								
d. Columns in tension	6	8	1.000	1	2.5	4	6	7	8

Table 7-11: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Braced Frames and Steel Shear Walls

Component/Action	$\frac{\Delta}{\Delta_y}$		Residual Force Ratio	Deformation					
				Primary				Secondary	
	d	e	c	IO	SE	LS	CP	LS	CP
Link Beam[3]									
a.[4] $\frac{2M_{CE}}{eV_{CE}} \le 1.6$	16	18	0.80	1.5	6.8	12	15	15	17
b. $\frac{2M_{CE}}{eV_{CE}} \ge 2.6$	Same as for beam in FR moment frame (see Table 7-28:)								
c. $1.6 \le \frac{2M_{CE}}{eV_{CE}} \le 2.6$	Use linear interpolation								
Steel Shear Walls[5]	15	17	.07	1.5	6.3	11	14	14	16

1. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
2. Δ_c is the axial deformation at expected buckling load.
3. Deformation is rotation angle between link and beam outside link or column. Assume Δ_y is 0.01 radians for short links.
4. Link beams with three or more web stiffeners. If no stiffeners, use half of these values. For one or two stiffeners, interpolate.
5. Applicable if stiffeners are provided to prevent shear buckling.

Table 7-11: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Braced Frames and Steel Shear Walls (Continued)

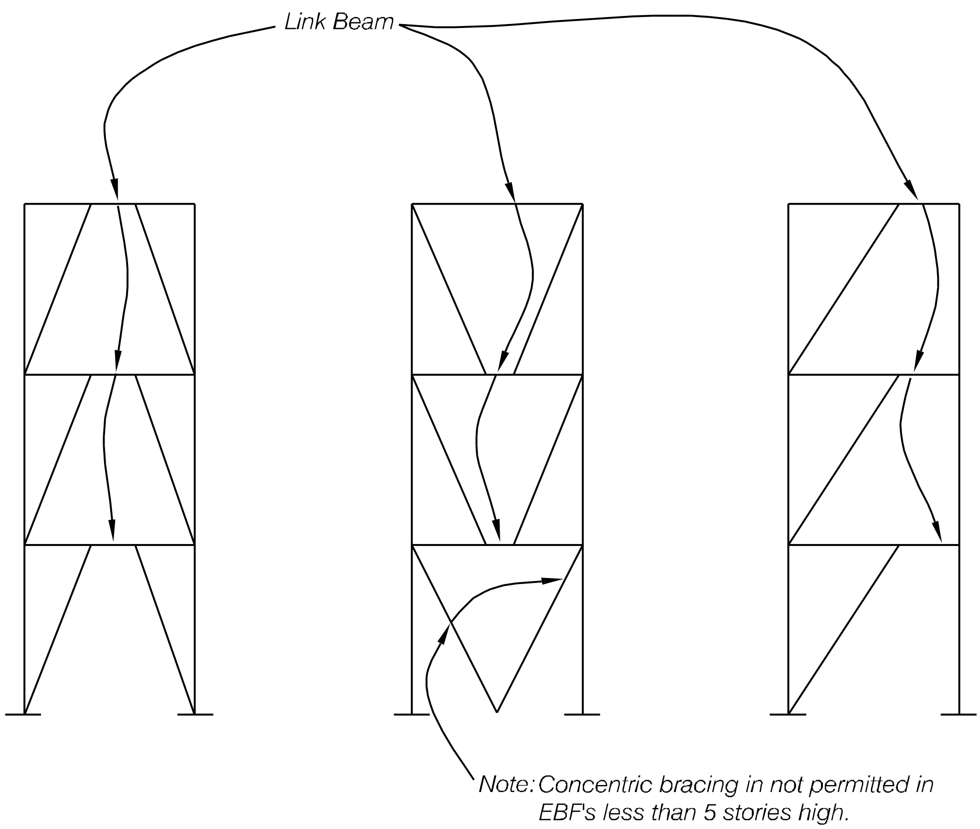


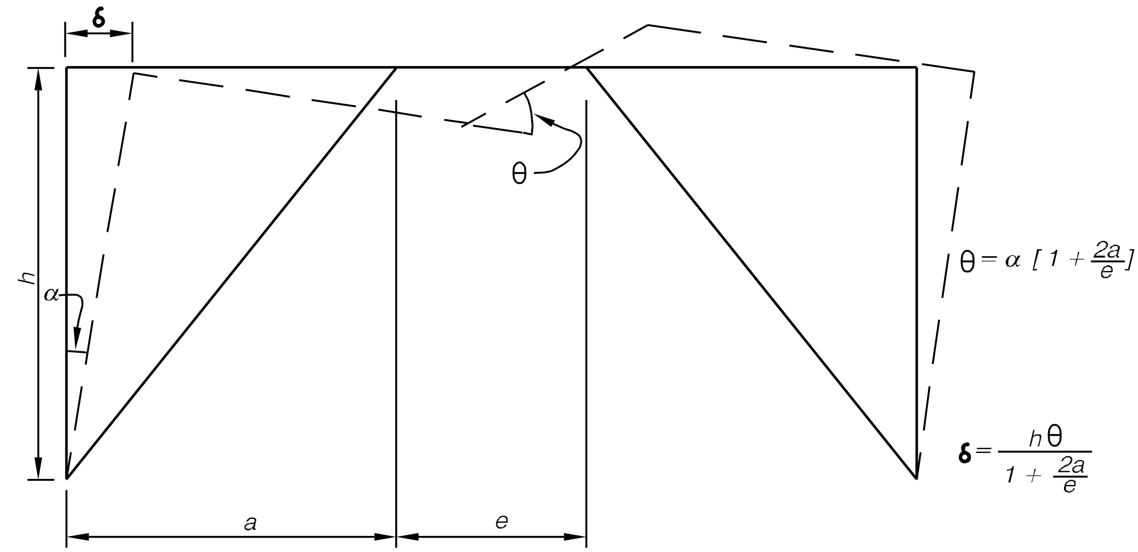
Figure 7-23 Eccentric braced frame configurations

a function of the geometry of the frame (see Figure 7-24). Link beams can yield in shear, in bending, or in both shear and bending at the same time. Which yield mechanism governs is a function of the relationship of link length to the ratio of its bending strength to shear strength. Where the length of the link beam is less than 1.6 M_s/V_s, the yielding is almost entirely in shear. Where the length is greater than 2.6 M_s/V_s, the yielding is primarily in bending. Where the length is between 1.6 M_s/V_s and 2.6 M_s/V_s, both shear and bending yield will occur. Since link beams that yield in shear are considered to have the most stable energy-dissipating characteristics, most of the EBF research has tested the cyclic inelastic capacity of link beams with shear yielding at large rotations. Consequently, most of the design provisions are concerned with limiting the link beam shear yield rotation to less than the maximum cyclic test rotations, and then requiring details indicated by the tests as necessary to ensure that this rotation can occur through a number of cycles without failure.

(4) Design criteria. The specific criteria governing the design of eccentrically braced frames are given in the AISC Seismic Provisions. Additional detail is provided in the following paragraphs.

(a) Link beam location and stability. Link beams are the fuses of the EBF structural system, and are to be placed at locations that will preclude buckling of the braces. A link beam must be located in the intersecting beam at least at one end of each brace. There are exceptions permitting concentric bracing at the roof level and/or at the bottom level of EBF over five stories in the AISC Seismic Provisions. Compact sections meeting the more restrictive flange-width-to-thickness ratio of $52/\sqrt{F_y}$ are required for the beam portions of eccentric braced frames in order to provide the beams with stable inelastic deformation characteristics. The same requirement is used for the beams of special moment-resisting space frames.

(b) Link beam strength. The basic requirement for link beam strength is given in the AISC Seismic Provisions, which states that the shear in the link beam web due to prescribed seismic forces be limited to 0.8 V_s. Paragraph 15.2f of the AISC Seismic Provisions addresses the concern for the effect that substantial axial loads in the link beam could have on its inelastic deflection performance. It presumes that in shear links, the web's capacity is fully utilized in shear, and that flanges provide the needed axial and flexural capacity. Shear links with a length less than 2.2 M_s/V_s are considered to be controlled by shear. Substantial axial loads occur in some EBF configurations when the link beam is required to transmit horizontal forces to or from the braces. It is recommended that, insofar as it is possible, link beams be located so that they are not required to transmit the horizontal force component of braces or drag struts. Where axial forces in the link cannot be avoided, the flexural strength shall be reduced by the axial stress f_a, giving $M_{RS} = Z(F_y - f_a)$. The f_a should correspond to the lesser value of the axial force corresponding to yield of the link beam in shear, or that which, when combined with link bending, causes the beam flanges to yield.



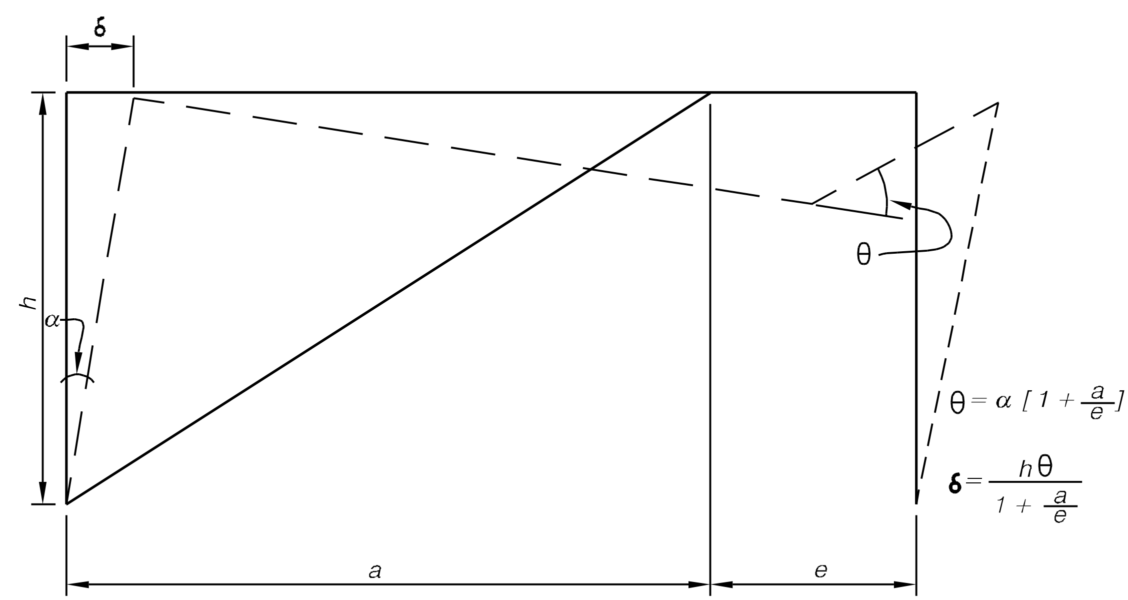


Figure7-24 Deformed frame geometry.

(5) Link beam rotation. The link beam rotation, at a frame drift 0.4*R* times the drift calculated from prescribed seismic forces, is limited to the values given in Paragraph 15.2g of the AISC Seismic Provisions. The procedure for calculating the rotations is as follows (refer to Figure 7-24):

(a) Perform an elastic analysis of the frame for the prescribed seismic forces, being certain that the analysis includes the contribution of the elastic shear deformation of the link beam.

(b) Calculate 0.4R times the drift angle obtained from the analysis in (1). This angle is denoted as ″ in Figure 7-24.

(c) Calculate the rotation angle *2*, as shown in Figure 7-24, for the appropriate configuration. This simplified procedure is slightly conservative, since the elastic curvature of the beam segments between hinges and of the brace deformations have been ignored, and would contribute a minor amount of the required deformation. It should be noted that calculation of the rotation by multiplying the elastic deflections of the link beam by 0.4*R* would be unconservative, since these deflections include elastic effects, such as the axial deformation of the braces, that would not increase proportionally after the link begins to yield.

(d) Link-beam web. Link-beam web doubler plates are prohibited in AISC Seismic Provisions because tests have shown that they are not fully effective. The performance of eccentric braced frames relies on the predictability of the strength and strain characteristics of the link beam. It is not considered advisable to complicate the behavior of the link beam by permitting doublers or allowing holes within it.

(e) Brace sizing. Once the link beam size has been selected, the brace size is determined by the requirement given in the AISC Seismic Provisions that its compressive strength be at least 1.5 times the axial force corresponding to the controlling strength of the link beam. The controlling strength is either the shear strength V_s or the reduced flexural strength M_{RS} described above, whichever results in the lesser force in the brace. Note that once the link beam is selected, the brace forces are determined from its strength, and the brace forces calculated in the elastic analysis will not govern, and will not be used in the brace design.

(f) Brace-to-beam connection. The AISC Seismic Provisions require that the brace-to-beam connection develop the compressive strength of the brace, and that no part of the brace-to-beam connection extend into the web area of the link. The required development may be at the strength level of the connection. The prohibition of the extension of the brace-to-beam connection into the link beam is intended to prevent physical attachments that might alter the strength and deflection characteristics of the link beam. It is not intended to prevent the centerline intersection of brace and link beam from intersecting within the link.

(g) Column sizing. FEMA 302 requires that the columns remain elastic at 1.25 times the forces causing yield of the link beam. “Remain elastic at” means the same as “have the strength to resist.” The strength, including bending moments, can be calculated using Part 2 of AISC "Specifications for Structural Steel Buildings."

(h) Beam-to-column connections. For link beams that are adjacent to a column, special connection criteria are given in Section 15.4 of the AISC Seismic Provisions. Where the link beam is not adjacent to the column, a simpler criterion for connection is given in Section 15.7 of the AISC Seismic Provisions. Where the simpler connections are used, consideration must be given to transmission of collector forces into the EBF bay.

(i) Intermediate stiffeners. Section 15.3 of the AISC Seismic Provisions provides requirements for various types of stiffeners necessary for the intended performance of the link beams. Stiffener plates as described in those paragraphs are required at the following locations (see Figure 7-25):

1. At the brace end(s) of the link beam.

2. At b_f from each end where link beam length is between 1.6 M_s/V_s and 2.6 M_s/V_s.

3. At intermediate points along the link beam where shear stresses control or are high.

(6) Acceptance criteria.

(a) Response modification factors, R, for Performance Objective 1A, are provided in Table 7-1.

(b) Modification factors, m, for enhanced performance objectives, are provided in Table 7-12 for beams, columns, and fully restrained moment connections; in Table 7-13 for partially restrained moment connections; and in Table 7-10 for braces and link beams.

(c) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided in Table 7-11 for deformation-controlled components.

(d) Expected strength of deformation-controlled components and lower-bound strength of force-controlled components shall be determined as indicated in Paragraph 7-3b(9).

7-4. Concrete Moment-Resisting Frames.

a. General.

(1) Function. Moment frames, like shear walls, are vertical elements in a lateral-force-resisting system that transmit lateral forces to the ground; however, they differ from shear walls in that their deflections result primarily from flexural deformations of their elements.

(2) Frame behavior. The bending stiffness of the moment-resisting frame provides the lateral stability of the structure (Figure 7-26). It is important to remember that deformations resulting from the dynamic response to a major earthquake are much greater than those determined from the application of the prescribed design forces. This

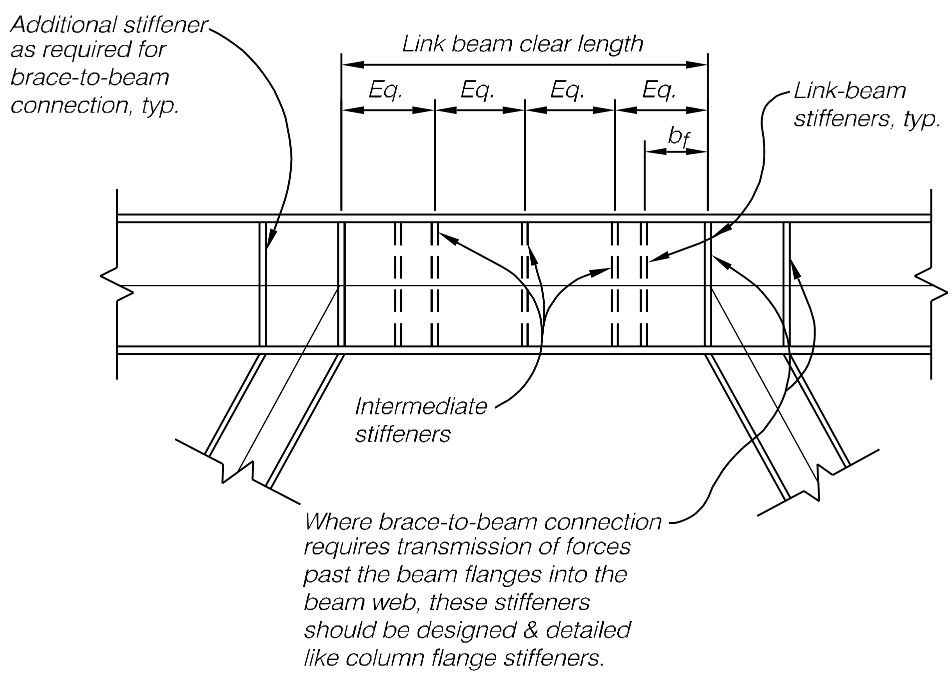


Figure 7-25 Link beam and intermediate stiffeners.

Component/Action	*m* Values for Linear Procedures[8]					
	Primary				Secondary	
	IO m	SE m	LS m	CP m	LS m	CP m
Moment Frames						
Beams:						
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	2	4	6	8	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1.5	2	3	3	4
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation						
Columns: For $P/P_{ye} < 0.20$						
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	2	4	6	8	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1	1	2	2	3
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation						
For $0.2 \le P/P_{ye} \le 0.50$[9]						
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	1	—[1]	—[2]	—[3]	—[4]	—[5]
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1	1	1.5	2	2
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation						
Panel Zones	1.5	4.8	8	11	NA	NA
Fully Restrained Moment Connections[7]						

1. $m = 6(1 - 1.7\,P/P_{ye})$
2. $m = 9(1 - 1.7\,P/P_{ye})$
3. $m = 12(1 - 1.7\,P/P_{ye})$
4. $m = 15(1 - 1.7\,P/P_{ye})$
5. $m = 18(1 - 1.7\,P/P_{ye})$
6. $m = 5 - 0.125\,d_b$
7. $m = 6 - 0.125\,d_b$
8. $m = 7 - 0.125\,d_b$
9. If construction documents verify that notch-tough rated weldment was used, these values may be multiplied by two.
10. For built-up numbers where strength is governed by the facing plates, use one-half these *m* values.
11. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

Table 7-12: Acceptance Criteria for Linear Procedures—Fully Restrained (FR) Moment Frames

Component/Action	*m* Values for Linear Methods					
	Primary				Secondary	
	IO	SE	LS	CP	LS	CP
Partially restrained moment connection						
For top and bottom clip angles[1]						
a. Rivet or bolt shear failure[2]	1.5	2.8	4	6	6	8
b. Angle flexure failure	2	3.5	5	7	7	14
c. Bolt tension failure[2]	1	1.3	1.5	2.5	4	4
For top and bottom T-stub[1]						
a. Bolt shear failure[2]	1.5	2.8	4	6	6	8
b. T-stub flexure failure	2	3.5	5	7	7	14
c. Bolt tension failure[2]	1	1.3	1.5	2.5	4	4
For composite top and clip angle bottom[1]						
a. Yield and fracture of deck reinforcement	1	1.5	2	3	4	6
b. Local yield and web crippling of column flange	1.5	2.8	4	6	5	7
c. Yield of bottom flange angle	1.5	2.8	4	6	6	7
d. Tensile yield of column connectors or OSL of angle	1	1.3	1.5	2.5	2.5	3.5
e. Shear yield of beam flange connections	1	1.8	2.5	3.5	3.5	4.5
For flange plates welded to column bolted or welded to beam[1]						
a. Failure in net section of flange plate or shear failure of bolts or rivets[2]	1.5	2.8	4	5	4	5
b. Weld failure or tension failure on gross section of plate	0.5	1.0	1.5	2	1.5	2
For end plate welded to beam bolted to column						
a. Yielding of end plate	2	3.8	5.5	7	7	7
b. Yield of bolts	1.5	1.8	2	3	4	4
c. Failure of weld	0.5	1.0	1.5	2	3	3

1. Assumed to have web plate or stiffened seat to carry shear. Without shear connection, this may not be downgraded to a secondary member. If $d_b > 18$ inches, multiply *m* values by $18/d_b$.
2. For high-strength bolts, divide these values by two.

Table 7-13: Acceptance Criteria for Linear Procedures—Partially Restrained (PR) Moment Frames

18 inches = 457mm

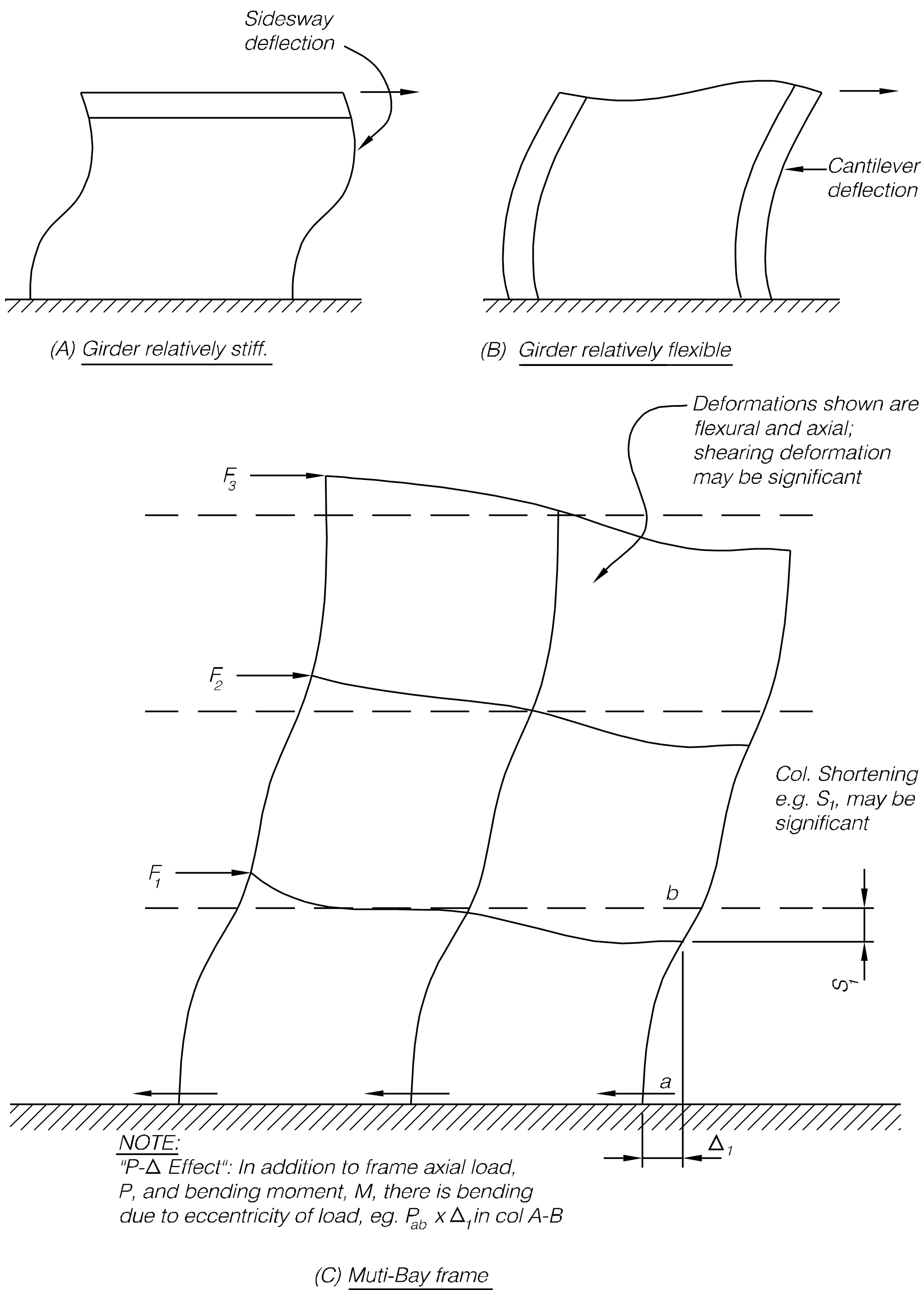


Figure7-26 Frame deformations.

means that a frame meeting the minimum strength requirements of this manual will survive a major earthquake only if it can yield and sustain cyclic inelastic deformations without essential loss of lateral resistance and vertical load capacity. Since normal building materials have very limited energy-absorbing capacity in the elastic range of action, it follows that what is needed is a large energy capacity in the inelastic range. The term "ductility" is used to denote this property. Providing a ductile seismic frame will allow the structure to sustain tolerable, and in many cases, repairable damage, instead of suffering catastrophic failure. The energy dissipation, ductility, and structural response (deformation) of moment-resisting frames depend upon the types of members, connections (joints), and materials of construction used. The behavior of joints is a critical factor in the ability of building frames to resist high-intensity cyclic loading.

(3) Mechanical and welded splices. See Paragraph 7-2a(3) for revisions to ACI 318 provisions regarding mechanical and welded splices in reinforcement.

b. Classification of Concrete Moment-Resisting Frames. FEMA 302 classifies concrete moment-resisting frames as Ordinary Moment Frames (OMF), Intermediate Moment Frames (IMF), or Special Moment Frames (SMF). Restrictions regarding the use of the various frame classifications are summarized in Table 7-1, which also provides the appropriate *R* value for each classification.

c. Nonseismic Frames. Frame members assumed not to contribute to lateral resistance shall be detailed according to Section 21.7.2 or 21.7.3 of ACI 318, depending on the magnitude of the moment induced in those members when subjected to the calculated displacements in FEMA 302. When the effects of lateral displacement are not explicitly checked, the provisions of Section 21.7.3 shall apply.

d. Ordinary Moment Frames (OMF) are reinforced concrete moment frames conforming to the provisions of ACI 318, exclusive of Appendix A.

(1) Flexural members of OMF's forming part of a seismic-force-resisting system shall be designed in accordance with Section 7.13.2 of ACI 318, and at least two main flexural reinforcing bars shall be provided continuously top and bottom throughout the beams through, or developed within, exterior columns or boundary elements.

(2) Columns of OMFs having a clear height-to-maximum plan dimension ratio of 5 or less shall be designed for shear in accordance with Section 21.8.3 of ACI 318.

e. *Intermediate Moment Frames (IMFs)* are frames conforming to the requirements of Sections 21.1, 21.2.1.1, 21.2.1.2, 21.2.2.3, and 21.8 of ACI 318, in addition to the requirements of OMFs. Flat-plate or two-way slabs are permitted for the beam elements of IMFs. These slab systems have a potential for a brittle mode of punching shear failure at the column supports due to gravity load combined with the eccentric shear caused by moment transferred from the slab to the column. In order to prevent punching shear failure under the maximum expected earthquake deformation, the slab shall be designed in accordance with Section 21.8 of ACI

318. Details illustrating these requirements are presented in Figures 7-27 through 7-32.

f. Special Moment Frames (SMFs) are frames conforming to the requirements of Sections 21.1 through 21.5 of ACI 318, in addition to the requirements of OMFs.

(1) General design requirements. The basic concept of SMFs is to provide inelastic energy dissipation by flexural yielding in the girder elements. Columns must, therefore, be stronger than the flexural capacity of the girders, and all elements must have shear resistance and reinforcing bar anchorage capacity capable of developing the full flexural yield level in the girders. In order to provide the girder yield mechanism, the design provisions require:

(a) Compact proportions for the girder and column sections, along with closely spaced seismic ties or hoops for confinement of concrete in the regions of potential flexural yielding.

(b) Column interaction flexural capacity greater than 6/5 times the value required to develop girder yield.

(c) Girder, column, and joint shear capacity greater than shears induced by gravity loads and the strain-hardened flexural capacity of the girders.

(d) Reinforcing bar splices and straight and hooked bar anchorages capable of developing the strain-hardened yield of the girder steel.

(e) Details illustrating the above requirements are presented in Figure 7-33 through 7-40.

(2) The two phases of design. With the design concept that inelastic behavior and energy dissipation are to be restricted to flexural yielding in the confined concrete regions of the beam or girder elements, the design process consists of two phases. The first phase establishes the beam sizes and capacities needed to resist the specified factored gravity and seismic load combinations. Then, with the known girder strengths and some preliminary column sizes, the second phase proportions the shear resistance of the girders, columns, and joints, and establishes the column flexural strengths such that all of these elements are able to resist the effects of a strain-hardened flexural yielding in the beams along with unfactored gravity loads.

g. Acceptance Criteria.

(1) Response modification factors, *R*, for Performance Objective 1A, for concrete frames in various structural systems are provided in Table 7-1.

(2) Modification factors, *m*, for enhanced performance objectives are provided in Table 7-14 for beams; Table 7-15 for columns; Table 7-16 for beam/column joints; and Table 7-17 for slab/column frames.

(3) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided in Table 7-18 for beams; in Table 7-19 for

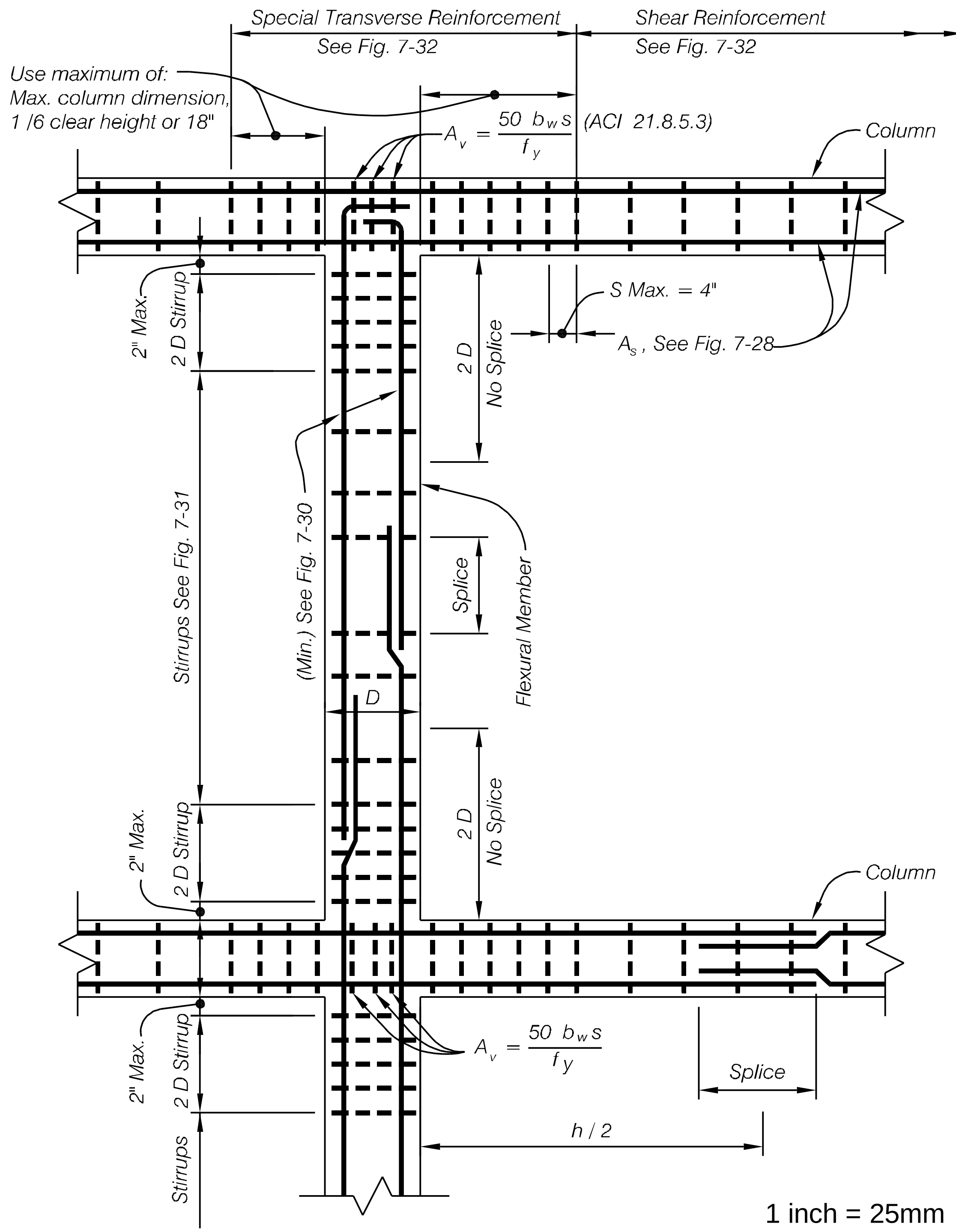
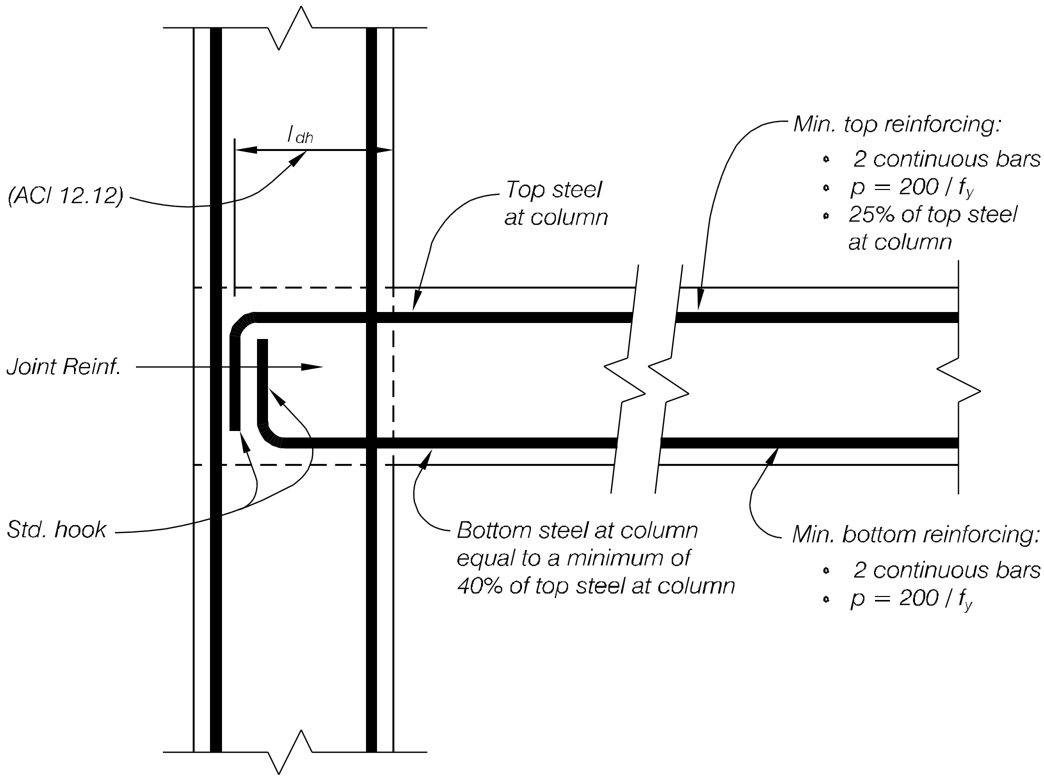


Figure 7-27 Intermediate Moment Frame frame requirements.



ELEVATION OF GIRDER AND COLUMN

FLEXURAL MEMBER:

f'_c = 3,000 p.s.i. min. at 28 days
f_y = 40 ksi or 60 ksi
Reinforcement ratio $p = A_S / bd$ or $p' = A'_S / bd$: $p = 0.025$ max.

1 ksi = 6.89 MPa

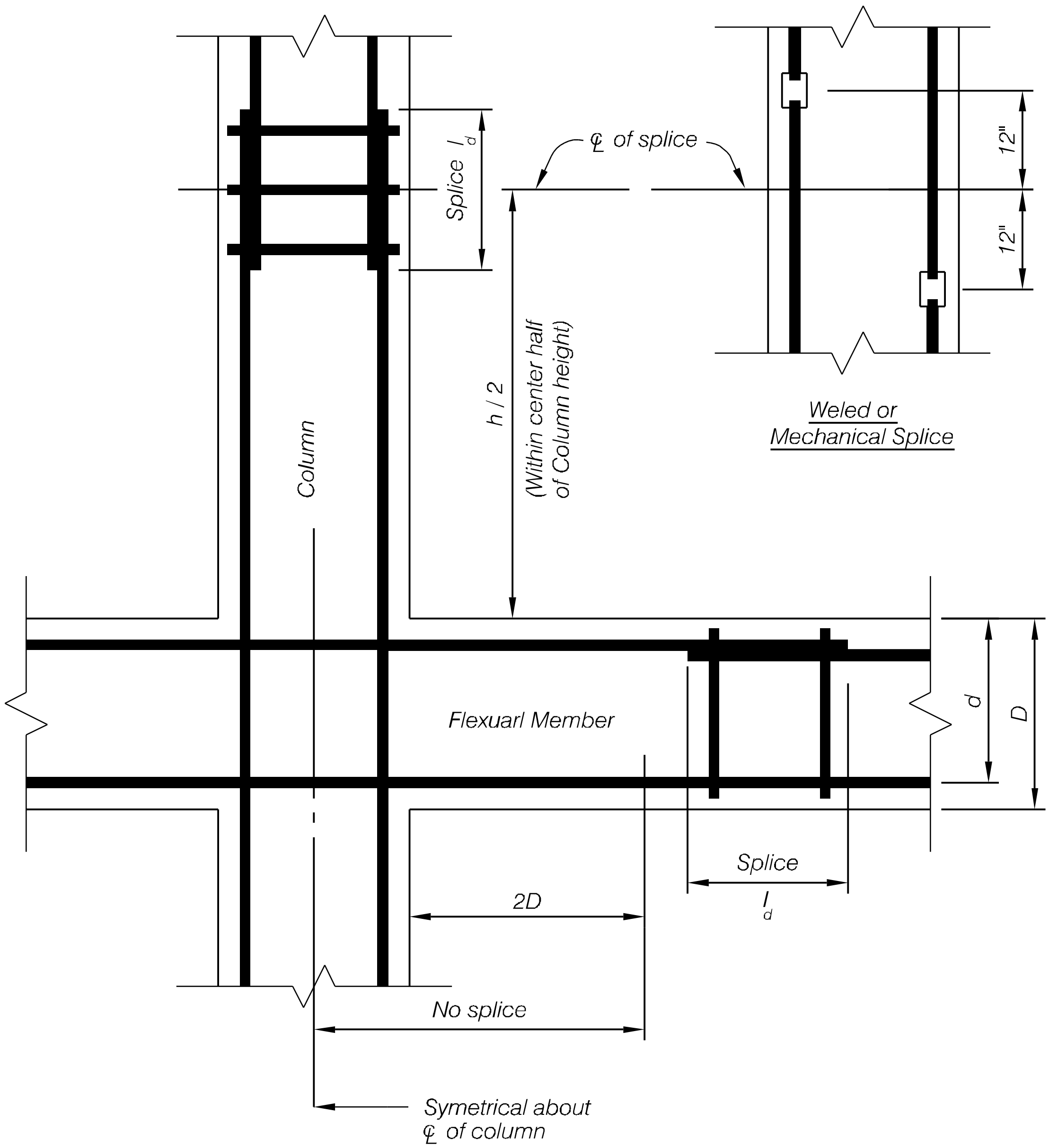
COLUMN:

f'_c = 3,000 p.s.i. min. at 28 days
f_y = 40 ksi or 60 ksi
Reinforcement ratio, p (for tied columns)

$$0.01 \leq p \leq 0.06$$

1 ksi = 6.89 Mpa

Figure 7-28 Intermediate Moment Frame longitudinal reinforcement



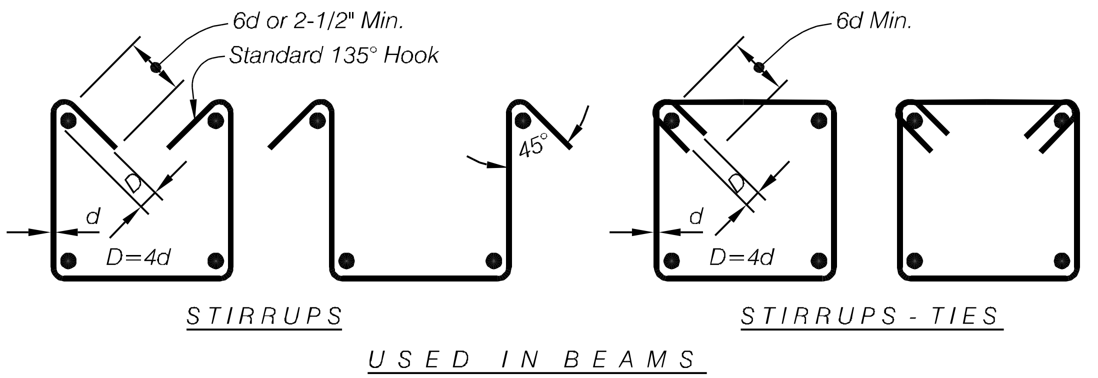
1 inch = 25mm

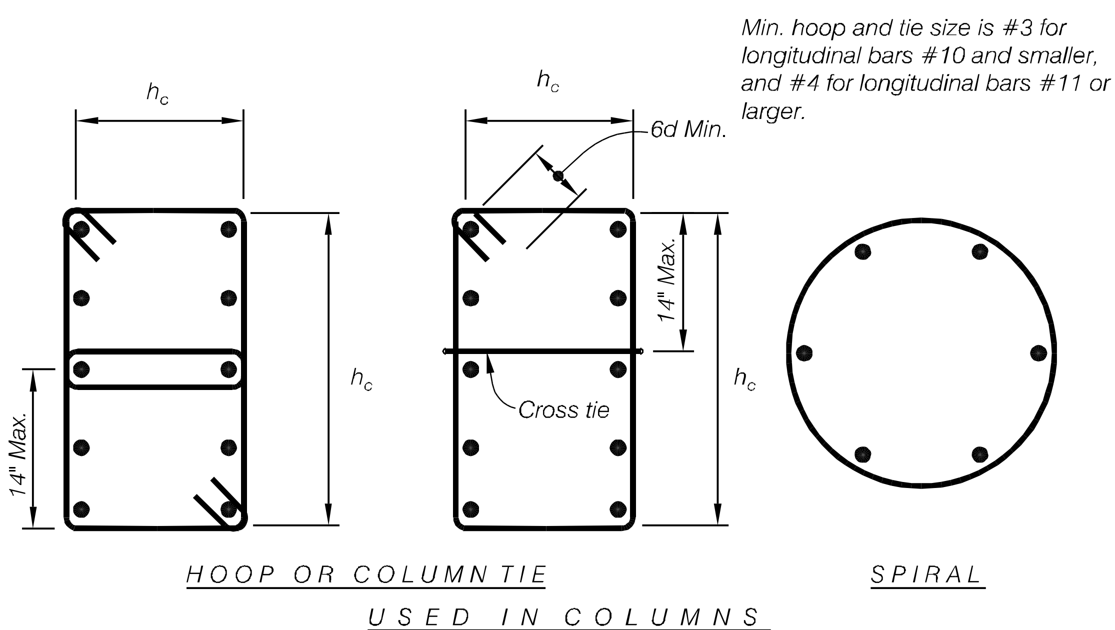
Column:

l_d Is the development length.

At any level, not more than alternate bars will be welded or mechanical spliced. Min. distance between two adjacent bar splices = 24".

Figure 7-29 Intermediate Moment Frame splices in reinforcement





Spiral Ratio:

$$\rho_s = 0.08 \frac{f'_c}{f_{yh}} \text{ or } 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$$

Whichever is greater.

Hoop Requirements -Total Tie Area:

$$A_{sh} = 0.08\ sh_c \frac{f'_c}{f_{yh}}$$

Functions	Stirrups	Stirrup-Ties	Column Ties	Hoops	Spirals
Shear Reinforcement and "Caging"	•	•	•	•	•
Restrain Longitudinal Steel from Buckling		•	•	•	•
Confine Concrete				•	•

1 inch = 25mm
#4 bar ≈ 10M bar
#9 bar ≈ 30M bar
#11 bar ≈ 35M bar

Figure 7-30 Intermediate Moment Frame transverse reinforcement